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Analysis and Design of Buried Steel Water Pipelines in Seismic Areas[1]

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ABSTRACT

The present paper offers an overview of available methodologies and provisions for the structural analysis and mechanical design of buried welded steel water pipelines subjected to earthquake action. Both transient (wave shaking) and permanent ground actions (from tectonic faults, soil subsidence, landslides and liquefaction-induced lateral spreading) are considered. In the first part of the paper, following a brief presentation of seismic hazards, modelling of the interacting pipeline-soil system is discussed, in terms of either simple analytical models or more rigorous finite elements, pin-pointing their main features. In the second part of the paper, pipeline resistance is outlined, with emphasis on the corresponding limit states. Possible mitigation measures for reducing seismic effects are also presented, and the possibility of employing gasketed joints in seismic areas is discussed. Finally, the above analysis methodologies and design provisions are applied in a design example of a buried steel water pipeline, located in an area with severe seismic action.

INTRODUCTION

The structural performance of steel water pipelines in geohazard areas is an issue of increasing interest. In the particular case of seismic action, the main purpose of pipeline operators is to minimize seismic risk on the pipeline, safeguarding the unhindered flow of water resources, following a severe seismic event. Towards this purpose, the structural damage of the steel pipe should be minimized, in order to maintain the structural integrity of the pipeline and prevent loss of water containment.

Earthquake actions in buried steel pipelines can be classified into two main categories: (a) transient actions, associated with wave shaking phenomena and (b) permanent ground-induced deformations, such as seismic faults, landslides, subsidence settlements, and liquefaction-induced

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lateral spreading. Past earthquakes have induced significant damage in buried pipelines, attributed to both transient and permanent ground deformations (Earthquake Engineering Research Institute (EERI, 1999); Liang and Sun, 2000; O'Rourke, 2003). These reports have indicated that damage due to permanent ground-induced deformations typically occurs in specific areas with severe ground motion, and is associated with high damage rates, whereas damages due to seismic wave action occur over substantially larger areas, but they are associated with lower damage rates.

The vast majority of research publications referring to the seismic analysis and design of buried steel pipelines has been driven by the need of safeguarding the integrity of hydrocarbon (oil and gas) pipelines. For transient ground-induced actions, the reader is referred to the paper by Kouretzis *et al.* (2006) for a more detailed literature review, whereas the more recent papers by Vazouras *et al.* (2010, 2012) provide a complete summary of previous works on permanent ground-induced actions on buried pipelines. Extensive experimental, analytical and numerical research on the effects of permanent ground-induced actions on the structural integrity of buried steel pipelines has been conducted in the course of the Safety of Buried Steel Pipelines Under Ground-Induced Deformations (GIPIPE) project (Karamanos *et al.*, 2015a; Vazouras *et al.*, 2015; Sarvanis *et al.*, 2016). In this research project, large-scale experiments have been performed, supported by extensive numerical simulations, whereas simple and efficient analytical methodologies have also been developed. It is worth noticing that current water pipeline design standards or manuals, such as American Water Works Association (AWWA M11), do not contain provisions for seismic design.

There exist several important differences between hydrocarbon and water pipelines, so that direct application of design guidelines and tools developed for oil and gas pipelines to water pipelines may not be appropriate. Steel water pipelines are different from hydrocarbon steel pipelines because they:

- are considerably thinner, with much higher values of D/t ratio
- are made of lower steel grade; X42 or X46 are usual grades for water steel pipes, whereas onshore hydrocarbon pipelines use X70 steel grade or higher.
- have different type of joints; oil and gas pipelines use almost exclusively butt-welded full-penetration joints, whereas water pipelines are constructed with welded-lap or gasketed joints.
- operate under lower pressure, which does not exceed 50% of yield pressure; this may not be necessarily beneficial, given the fact that, in most case, the presence of internal pressure may prevent cross-sectional distortion, increasing pipeline deformation capacity.

- contain special components (e.g. elbows and junctions) with have a different geometry and configuration than the corresponding components in oil & gas pipelines.

The main seismic design requirement is that pipeline seismic actions should be less than the corresponding pipeline resistance. The present paper offers an overview of seismic analysis and design of buried welded steel pipelines for water transmission and distribution, based on existing information in the literature and in relevant codes, standards and design guidelines. Following an outline of existing provisions in pipeline design standards and recommendations in North America and Europe, the paper refers to seismic actions, due to both transient and permanent ground deformations. In the second part of the paper, issues related to pipeline resistance are presented, with direct reference to possible failure modes. Possible measures for mitigating seismic effects on buried pipelines are also discussed. Finally, a design example that illustrates the application of the above methodologies and design provisions is presented.

EXISTING STANDARDS AND RECOMMENDATIONS FOR PIPELINE SEISMIC DESIGN

The American Society of Civil Engineers (ASCE, 1984) guidelines have been the first document that transferred and adjusted existing knowledge and design tools of seismic engineering into the earthquake analysis and design of buried pipelines. In particular, the document has been based mainly on the relevant work by N. M. Newmark, W. J. Hall and their associates at the University of Illinois (e.g. Newmark, 1967; Newmark and Hall, 1975). This document has also been the basis for the American Lifelines Alliance (ALA, 2005) guidelines, which contains the most complete set of provisions for this subject. Some of the ALA (2005) provisions will be used in the present paper. This research work has also constituted the basis for the recent Indian National Information Center of Earthquake Engineering (NICEE) guidelines (2007) for earthquake design of buried pipelines.

The Pipeline Research Council International (PRCI, 2004) guidelines for the pipeline earthquake design and assessment can be considered as an update of the ASCE (1984) guidelines for buried pipelines transporting natural gas and liquid hydrocarbons. In particular, they accounted for more recent research on soil loading on buried pipelines, on strain-based pipeline limit states, and proposed more advanced tools for pipeline stress analysis. More recently, PRCI has published design guidelines for the design of oil and gas guidelines in landslide areas (PRCI, 2009), which adopt analysis and design methodologies similar to the ones proposed in PRCI (2004).

American Society of Mechanical Engineers (ASME) B31.4 and ASME B31.8 standards, widely used for oil and gas pipeline design respectively, state that earthquake loading should be considered in pipeline design as an accidental (environmental) load. Nevertheless, those standards do not contain information on how seismic action on the pipeline should be computed. Similarly, Canadian Standard Association (CSA) Z662 specifies fault movements, slope movements, and seismic-related earth movements as additional loading that should be taken into account for pipeline design, but does not provide any further information on how those actions should be quantified.

European standard Comité Européen de Normalisation (EN) 1594 has been a popular standard for the general design of high-pressure gas pipelines. Annexes D and E of this standard refer to landslide and high-seismicity areas respectively; in both Annexes, it is suggested that these geohazards should be taken into account in pipeline analysis and design, whereas some mitigation measures are also proposed. Similarly, European standard EN 16416, also known as International Organization for Standardization (ISO) 13623 standard, in subsection 6.3.3.3 provides general information and suggestions on seismic design. European standard EN 1998-4, provides guidance for the earthquake analysis and design of buried pipelines. One should notice that this standard has been developed primarily for the seismic design of liquid storage tanks, whereas limited information on buried pipelines is contained in Chapter 6 and Annex B. Furthermore, EN 1998-4 is intended to cover all possible materials (steel, concrete, plastic), and therefore, it may not be a standard suitable for the seismic design of buried steel pipelines. However, some clauses of EN 1998-4 can be useful for pipeline design and will be employed in the present paper. Finally, among numerous national standards for pipeline design, the Dutch standard Nederlands Normalisatie-Instituut (NEN) 3650 is highlighted; despite the fact that earthquake action may not be an issue in The Netherlands, NEN 3650 contains important information for ground-induced actions on pipelines, especially for soil-pipe interaction in settlement areas.

SEISMIC ACTIONS IN CONTINUOUS BURIED PIPELINES

Ground-induced earthquake actions on buried pipelines, can be categorized in (a) transient actions and (b) permanent deformations. Transient actions are caused by wave shaking effects, whereas permanent ground deformations are due to fault movements, landslide activation and liquefaction-induced lateral spreading. The present section examines the effects of ground-induced earthquake actions on continuous steel buried pipelines. Those are welded pipelines with welded-lap joints, whereas butt-welded connections are employed only in few instances.

Transient action

Transient action is often referred to as "wave propagation hazard", and is characterized by peak ground acceleration and velocity, as well as the appropriate propagation velocity. It is caused by ground shaking due to body and surface seismic waves travelling within the soil. Body waves (compressional and shear) propagating through the three-dimensional ground, are generated by seismic faulting at the seismic source. Surface waves (Love and Rayleigh) travel along the ground surface, and are generated by the boundary condition imposed by ground surface to body waves.

Seismic wave action analysis of a buried pipeline is a complex problem requiring wave propagation analysis on the three-dimensional soil-pipe system, accounting for the soil-pipe interface. As an alternative, the simplified method developed by Newmark (1967) can be employed, which estimates soil strain and curvature due to a traveling wave of constant shape, in terms of peak ground motion parameters. In this method, the maximum ground strain ε_g in the direction of wave propagation can be expressed by the following equation:

$$\varepsilon_g = \frac{PGV}{C} \tag{1}$$

where PGV (Peak Ground Velocity) is the maximum horizontal ground velocity in the direction of wave propagation and C is the apparent velocity of the seismic wave. The maximum axial force on the pipeline can be computed as the minimum value of F_1 and F_2, defined as follows (ALA, 2005):

$$F_1 = EA\varepsilon_g \tag{2}$$

and

$$F_2 = (t_u \lambda)/4 \tag{3}$$

where t_u is the ultimate frictional force of soil per unit pipe length, acting on the pipe in the axial direction and λ is the corresponding wavelength at pipe location. In addition, the maximum ground curvature, k_g, can be computed as the second derivative of the transverse displacement with respect to the axial coordinate along the pipe, resulting in the following equation:

$$k_g = \frac{PGA}{C^2} \tag{4}$$

where PGA(Peak Ground Accelaration) is the maximum ground acceleration perpendicular to the direction of wave propagation. The peak ground motion parameters PGV and PGA can be obtained from seismic records available in the area of interest, from relevant seismic maps or following a dedicated local geotechnical analysis. Furthermore, there exist several arguments on the choice of the value of C in equations (1) and (4). For a site subject to body wave propagation, the value of C should be taken in the range of 2,000 to 5,000 m/sec, as noted by O'Rourke and El Hmadi (1988), whereas the ALA guidelines (2005) suggest, a value equal to 4,000 m/sec (13,000 ft/sec), which is within the above range. However, in certain cases where Rayleigh waves are important, it may be necessary to consider lower propagation velocities, typically as low as 500m/sec, which is a lower (conservative) bound for the apparent (effective) wave velocity, as reported in the analysis of Trifunac and Lee (1996). This is in agreement with the peak ground strain versus PGV data reported by Iwamoto *et al.* (1998) and Paolucci and Smerzini (2008), also summarized by O'Rourke and Liu (2012). These data show that the lowest inferred C values lie between 500 and 1000 m/sec It should be noticed that these low C values are representative for low velocity-soft sedimentary formations (e.g. Holocene sediments), whereas higher values should be employed for sites located on stiffer formations (e.g. older sediments, bedrocks, etc.).

Permanent ground actions – analytical methods

A significant number of earthquake damages to steel pipelines have been attributed to permanent ground deformations (fault movements, landslides, soil subsidence and liquefaction-induced lateral spreading). Permanent ground deformations are applied on the pipeline in a quasi-static manner, and they are not necessarily associated with severe seismic events; however, under those actions the pipeline may be seriously damaged.

Fault movement

An active tectonic fault constitutes a discontinuity between two portions of the bedrock, along which relative motion of the two portions may occur. An active tectonic fault is a planar fracture or discontinuity in a volume of rock, across which significant displacement may occur as a result of earth movement. The movement is concentrated in a rather narrow fault zone and can be horizontal (strike-slip fault) or vertical (normal or reverse fault) as shown in Figure 1 and also can be an oblique direction (oblique fault). It is possible to estimate fault displacement PGD_F (Peak Ground Displacement) in terms of earthquake moment magnitude using empirical relations, such as the equations proposed by Wells and Coppersmith (1994).

Subsequently, the axial strain induced in the pipeline wall can be estimated, using the analytical procedure developed by Kennedy *et al.* (1977). For the case of horizontal (strike-slip) faults (Figure 2a), which employs the horizontal ground-induced displacement PGD_{FH}. According to this methodology, the axial strain caused by pipeline stretching ε_m, referred to as "membrane strain", can be computed as follows:

$$\varepsilon_m = \frac{PGD_{FH}}{L_H}\cos\theta + \frac{2}{3}\left(\frac{PGD_{FH}}{L_H}\sin\theta\right)^2 \qquad (5)$$

where θ is the angle between the fault plane and the pipeline axis, L_H is the distance between the two ends of the S-shaped pipeline configuration. The first term is linear and is due to the fault motion component in the direction of the pipeline axis. The second term is quadratic due to axial stretching because of pipeline transverse deformation.

A similar equation exists in ALA Guidelines (2005):

$$\varepsilon_m = 2\frac{PGD_{FH}}{L_H}\cos\theta + \left(\frac{PGD_{FH}}{L_H}\sin\theta\right)^2 \qquad (6)$$

Comparison of equations (5) and (6) indicates that the latter contains an additional factor of 2, which is aimed at accounting for the uncertainties of the methodology of Kennedy *et al.* (1997). It is important to notice that axial deformation of the pipeline extends well beyond the S-shape pipe segment, and that the above equations (5) and (6) refer only to axial deformation (stretching) of the S-shape of the pipe.

For an oblique fault with fault movement PGD_{FV} in the vertical direction and PGD_{FH} in the horizontal direction, one may write the following equation for the axial strain in the pipeline,

$$\varepsilon_m = \frac{PGD_{FH}}{L_H}\cos\theta + \frac{2}{3}\left(\frac{PGD_{FH}}{L_H}\sin\theta\right)^2 + \frac{2}{3}\left(\frac{PGD_{FV}}{L_V}\right)^2 \qquad (7)$$

where L_V is the distance between the two ends of the S-shaped pipeline configuration, shown in Figure 2b. A deficiency of the above analytical methodologies is that they do not provide a reliable methodology for determining the values of L_H and L_V.

A more elaborate, yet very efficient, analytical methodology for determining the strain in buried pipelines at fault crossings, has been presented by Sarvanis and Karamanos (2016). This methodology employs an assumed shape function, it is applicable to both horizontal and normal faults, and provides a systematic procedure for the calculation of lengths L_H and L_V in terms of soil conditions.

Furthermore, it is important to underline that equations (5) and (7) refer only to pipeline stretching, and neglect pipeline bending resistance, which can be important. For the case of a normal fault, with fault movement PGD_{FV} in the vertical direction, an analytical expression for the maximum bending strain is proposed in the analytical methodology of Sarvanis and Karamanos (2016), as follows:

$$\varepsilon_b = \left(\frac{\pi^2}{8}\right)\frac{D}{L'_V L_V}\left(PGD_{FV}\right) \qquad (8)$$

where L_V' is the distance from the end of the S-shape configuration to the inflection point (Figure 2b).

Finally, one has to notice that the above analytical equations should be used in cases where the pipeline alignment in the fault area is straight, without bends. Bends are significantly more flexible with respect to straight pipes, and exhibit significant stress and strain concentrations. The presence of bends near the fault zone may affect significantly pipeline stress and strain; in such a case, the above analytical expressions for strain may not provide reliable predictions, and the use of a numerical finite element model is recommended for pipeline analysis.

Landslides

Landslides are associated with massive ground movements caused by soil slope instability (Figure 3a). The primary driving force for a landslide is soil gravity, but a seismic event may trigger this phenomenon. Numerous empirical methodologies have been reported to determine the occurrence a landslide in terms of the distance from the epicentre and the magnitude of the earthquake event. To quantify the effects of landslide on pipeline deformation, the expected landslide movement PGD_S is required, and this can be estimated by available analytical expressions (Jibson, 1994).

In the case of permanent ground-induced action in the longitudinal direction due to landslide, the pipeline should be designed for an axial force F, which is the minimum of F_1 and F_2, expressed in the following equations proposed by ALA guidelines (2005):

$$F_1 = \sqrt{EAt_u\left(PGD_S\right)} \qquad (9)$$

and

$$F_2 = \left(t_u L_S\right)/2 \qquad (10)$$

In the above equations, t_u is the maximum (ultimate) frictional force of soil per unit pipe length acting on the pipe in axial direction, and L_S is the length of pipe in soil mass undergoing movement. According to ALA (2005), the value of L_S may range between 100 and 250 meters.

In the case of permanent landslide action in the transverse direction, the bending strain in the pipeline can be estimated by the following expression, assuming a cosine function of the pipe deformation:

$$\varepsilon_b = \frac{\pi^2 D\left(PGD_S\right)}{W^2} \qquad (11)$$

where W is the width of the landslide zone, ranging between 150 and 300 meters, according to ALA (2005). Alternatively, assuming a beam with both ends fixed and a uniform lateral load p_u one readily obtains equation (12) for the bending strain:

$$\varepsilon_b = \frac{p_u W^2}{3\pi EtD^2} \tag{12}$$

It is also noted that transverse permanent ground actions induce also axial tensile strains due to pipeline stretching.

Lateral spreading

Lateral spreading is a consequence of liquefaction in a sandy soil layer; the soil looses its shear strength, resulting in lateral movement of the liquefied soil, primarily in the horizontal direction (Figure 3b). In liquefaction-induced lateral spreading, if the pipeline is contained in the liquefied layer, buoyancy should be taken into account, together with the horizontal ground movement imposed to the pipeline. To estimate permanent ground displacement due to liquefaction PGD_L, several methodologies have been proposed (e.g. Bardet et al., 2002). For longitudinal action, the corresponding maximum axial force in the pipeline can be calculated through equations (9)-(10), whereas for transverse lateral-spreading action, the maximum bending strain can be computed from equations (11)-(12), replacing PGD_S with PGD_L.

Permanent ground deformation – finite element modelling

Finite element modelling is a more rigorous tool for simulating the effects of ground-induced actions on a buried pipeline. The finite element analysis of buried pipelines requires some computational effort and expertise, but offers an advanced tool for determining stresses and strains within the pipeline wall with significant accuracy with respect to the analytical formulae described above. There exist two levels of finite element modeling, briefly described below. Level 1 is adequate for regular design purposes, whereas level 2 is used only in special cases, where increased accuracy is necessary.

Level 1: beam-type finite element analysis

In this type of finite element analysis, the pipe is modelled with beam-type one-dimensional finite elements. These models have been used mainly for simulating permanent ground-induced actions on pipelines, but it can be used for modelling wave effects as well. The finite element mesh near discontinuities (e.g. fault plane) should be fine enough, so that gradients of stress and strains are accurately described (Figure 4a).

Type of finite elements: The use of regular beam elements for the pipeline model is not recommended, because they cannot account for pressure. Instead, "pipe elements" are preferable for pipeline seismic analysis. These are enhanced beam-type elements that account for the effect of hoop stress due to pressure. However, "pipe-elements" usually have a circular cross section and do not describe cross-sectional ovalization. Therefore, the use of more elaborate "pipe elements", capable of describing cross-sectional ovalization, sometimes referred to as "elbow elements", can further improve the accuracy of the finite element model, especially at pipe bends (Bathe and Almeida. 1982; Karamanos and Tassoulas, 1996). Alternatively, it is possible to employ regular pipe elements, which are essentially beam elements with circular cross-section, accounting for ovalization effects at pipe bends through the use of appropriate flexibility factors, and stress intensity factors.

Pipe and soil modelling: Pipe material should be modelled as elastic-plastic, considering strain-hardening. The ground surrounding the pipeline should be modelled by nonlinear springs (Figure 4a), attached to the pipe nodes and directed in the transverse directions (with stiffness k_V and k_H in the vertical and lateral direction respectively) and axially (k_{ax}). The springs should account possible slip between the pipe and the soil. Expressions for these soil stiffness are offered in ALA (2005), based on the type of soil. Alternative expressions for those springs can also be found in the NEN 3650 standard. The reader is also referred to the recent works of Xie *et al.* (2013) and Saiyar *et al.* (2016) regarding the limitations of soil spring reaction models, especially for the case of flexible pipes. In addition, comparison of "pipe" and "elbow" element methodologies with more rigorous finite element methodologies and experimental data have been reported recently by Sarvanis *et al.* (2016) and Sarvanis and Karamanos (2016).

Analysis procedure and output: To perform pipeline analysis under permanent ground-induced actions, the imposed soil displacements should be applied at the ends of the soil springs. The analysis follows three steps: (a) gravity, (b) operational loading (pressure and temperature) and (c) PGD application. The analysis output consists of stress resultants in pipeline cross-sections, as well as the stresses and strains in the longitudinal direction. The user should be cautioned that if the finite elements are not capable of describing accurately cross-sectional distortion the stresses and strains obtained may be quite different than the real stresses and strains in the pipeline wall, especially when the pipe wall begins to wrinkle due to local buckling. Consideration of local stresses due to pipe wall wrinkling locations requires a more detailed analysis, with the use of shell elements for modelling the pipe.

Level 2: three-dimensional finite element analysis

Three-dimensional finite element models constitute a rigorous numerical tool to simulate buried pipeline behavior under PGD. Such a model can describe in a rigorous manner the nonlinear geometry of the deforming soil-pipe system (including distortions of the pipeline cross-section), the inelastic material behavior for both the pipe and the soil, as well as the interaction between the pipe and the soil. However, it requires computational expertise.

Discretization: an elongated prismatic model is considered, where the steel pipeline is embedded in the soil, as shown in Figure 4b for the case of a strike-slip fault. Shell elements are employed for modeling the steel pipeline segment, whereas three-dimensional "brick" elements are used to simulate the surrounding soil. The discontinuity plane (e.g. fault plane, edge of landslide or lateral spreading) divides the soil block in two parts. The analysis is conducted in three steps; gravity loading is applied first, followed by the application of operation loads and, finally, the ground-induced movement is imposed holding one soil block fixed, an imposing a displacement pattern in the external nodes of the second block. A fine mesh should be employed at the part of the pipeline where maximum stresses and strains are expected. Similarly, the finite element mesh for the soil should be more refined in the region near fault and coarser in the region away from the fault. The relative movement of the two blocks is considered to occur within a narrow zone of width w to avoid numerical problems.

Material models: the constitutive models should account for the elastic-plastic behavior of both the pipeline and soil. Von Mises plasticity with isotropic hardening can be employed for describing pipe steel material, calibrated through a uniaxial stress-strain curve from a tensile test. Furthermore, an elastic-perfectly plastic Mohr-Coulomb model can considered for modelling soil behavior. This model is characterized by the soil cohesiveness c, the friction angle ϕ, the elastic modulus E, and the Poisson's ratio ν. Furthermore, a contact algorithm should be employed to simulate the interface between the outer surface of the steel pipe and the surrounding soil, taking into account interface friction, and allowing separation of the pipe and the surrounding soil.

Analysis procedure and output: it is suggested that the analysis should follow a displacement-controlled scheme, which increases gradually the ground displacement. At each increment of the nonlinear analysis, stresses and strains at the pipeline wall should be recorded. Furthermore, using a fine mesh at the critical pipeline portions, local buckling (wrinkling) formation and post-buckling deformation at the compression side of the pipeline wall can be simulated in an explicit manner.

SEISMIC RESISTANCE OF STEEL PIPELINES

Pipeline performance criterion and limit states

In pipeline seismic design, the main target is pipeline integrity against loss of containment. One should notice that a severe seismic event may cause significant deformation of the pipeline, well beyond the elastic regime of the pipe steel material, so that traditional pipeline design based on allowable stress may not be applicable. Therefore, the corresponding performance criterion can be stated as "pipeline may exhibit damage, but should maintain its water containment, so that it continues to fulfil its operational function after the seismic event".

There exist several limit states for continuous (welded) pipelines:

- Pipe wall fracture due to excessive tensile strain (base material and butt-welded joints)
- Pipe wall local buckling due to excessive compressive strain
- Pipeline overall buckling due to compressive loading
- Failure of welded-lap joints (fracture or crushing) and pipe components

In the course of a pipeline earthquake design procedure, the failure modes are quantified in terms of strain and deformation capacity, as described in the following.

Maximum tensile strain capacity

Exceedance of tensile strain capacity may cause fracture of pipeline wall. In the absence of serious defects or damage in the pipeline, the tensile capacity is governed mainly by the strength of the pipeline field welds, which are usually the weakest locations due to weld defects and stress/strain concentrations. Tensile strain limits of butt welds are experimentally determined through appropriate tension tests on strip specimens and on wide plates (Wang *et al.*, 2010). In several standards and guidelines, the suggested value of the ultimate tensile strain ε_{Tu} for butt-welded water pipelines varies between 2% and 5%. The value of 3% for tensile strain limit is adopted by the EN 1998-4 provisions for seismic-fault-induced action on buried steel pipelines, however, it is not clear whether it is applicable to welded lap joints. ALA (2005) limits for tensile strain are very similar, suggesting a limit strain equal to 2% for double-welded lap joints. PRCI (2004) suggest, for the case of oil and gas pipelines, a limit value within 2%-4% for pressure integrity and a limit within 1%-2% for normal operability. Finally, Annex C of CSA Z662 pipeline design standard provides an equation for calculating tensile strain limit ε_{Tu} of pipeline girth welds, considering surface defects. One should note that the above limit values for the maximum tensile strain ε_{Tu} refer to the "macroscopic" strain calculated from a stress analysis methodology, as described in the previous sections of this paper; this value of strain is quite different than the strain in the vicinity of the weld toe.

Local buckling

Compressive ground-induced strains may also occur due to axial compression and pipe bending deformation. When compressive strains exceed a certain limit, pipeline wall becomes structurally unstable, and fail in the form of local buckling or wrinkling, as shown in Figure 5a (see also Van Es *et al.*, 2016; Vasilikis *et al.*, 2016). Initially, despite the presence of those "wrinkles" or "buckles", the pipeline may still fulfill its basic function (i.e. water transmission), provided that the steel material is adequately ductile (Gresnigt, 1986). However, the buckled area is associated with significant strain concentrations and, in the case of repeated loading due to operation conditions (e.g. rather small variations of internal pressure or temperature), fatigue cracks may develop, imposing serious threat for the structural integrity of the pipeline (Dama *et al.*, 2007, Pournara *et al.*, 2015). Compressive strain limits for steel pipes depend primarily on the diameter-to-thickness ratio (D/t) and the level of internal pressure, and secondarily on the yield stress of steel material σ_y. Initial imperfections and residual stresses (as a result of the manufacturing process) may also have a significant effect on the critical compressive strain (Gresnigt and Karamanos, 2009). The value of local buckling (ultimate compressive) strain ε_{Cu} can be estimated using the following design equation, initially proposed by Gresnigt (1986), adopted by NEN 3650 and CSA Z662:

$$\varepsilon_{Cu} = 0.5\left(\frac{t}{D}\right) - 0.0025 + 3000\left(\frac{\sigma_h}{E}\right)^2 \tag{13}$$

where the hoop stress σ_h depends on the level of internal pressure p:

$$\sigma_h = \begin{cases} p(D/2t), & if \quad p(D/2t) \le 0.4\sigma_y \\ 0.4\sigma_y, & if \quad p(D/2t) > 0.4\sigma_y \end{cases} \tag{14}$$

Another equation for the ultimate buckling strain has been proposed by DNV-OS-F101 standard:

$$\varepsilon_{Cu} = 0.78\left(\frac{t}{D} - 0.01\right)\left(1 + 5.75\frac{p}{p_b}\right)\alpha_h^{-1.5}\alpha_{gw} \tag{15}$$

where p_b is the burst pressure, α_h is a hardening factor that depends on the yield-to-tensile strength (Y/T) ratio, and α_{gw} is a girth weld factor, given the fact that this equation has been proposed for girth-welded pipes.

Beam buckling

Under excessive quasi-uniform compressive loading, the pipeline may buckle as a beam. The pipeline is very long with respect to its cross-section, which means that it is very slender. Therefore, the main resistance parameter against beam buckling is the lateral resistance offered by the surrounding soil. This implies that shallow trenches and/or backfills with loose materials may result in the activation of this failure mode. In general, beam buckling load is an increasing function of the cover depth and the stiffness of the backfill material. Hence, if a pipe is buried at a sufficient depth, it will develop local buckling before the occurrence of beam buckling. To design water pipelines against beam buckling, one may use the design tools for the design of high pressure – high temperature oil and gas pipelines against beam-buckling, referred to as "upheaval" or "thermal" buckling (Palmer and King, 2008), or employ the nomographs proposed by Meyersohn (1991), also reported by O'Rourke (2003), which provide the critical cover depth of a buried pipeline. It should be noted that this failure mode is more likely to occur in oil and gas pipelines, where significant axial compression may develop due to pressure and temperature. On the other hand, water pipelines may develop high compression in the case of a permanent ground-induced action, mainly when loaded in the direction of the pipeline axis, and therefore, this mode should be considered in the course of an earthquake design procedure.

Distortion of pipeline cross-section

To maintain the pipeline operational, it is necessary to avoid significant distortions of the pipeline cross-section. This is more likely to occur in low-pressure thin-walled pipelines, whereas internally pressurized pipelines exhibit less cross-sectional distortion due to the stabilizing effect of internal pressure. This is a serviceability limit state, not related directly to failure and loss of containment, and a simple measure of cross-sectional distortion is the non-dimensional "flattening parameter" f defined in terms of the ratio of the maximum change of pipe diameter ΔD over the original diameter D (see also Figure 5b):

$$f = \Delta D / D \tag{16}$$

Following Gresnigt (1986) and NEN 3650, a cross-sectional flattening limit state is reached when the value of f becomes equal to 0.15.

Resistance of pipeline joints and fittings

Welded-lap pipe joints offer a simple and efficient way for connecting large-diameter thin-walled line pipes. The weld can be external, internal or at both sides. The eccentricity of longitudinal stress path along the pipeline at this connection, together with the fillet-type weld, may result in a reduction of pipe joint strength with respect to the strength of the line pipe itself. Furthermore, welded lap joint efficiency also depends on the ratio l/t where l is the length of the curved portion of the female (O'Rourke and Liu, 2012). Limited work has been published on the response of those joints under severe structural loading. The tensile capacity of welded-lap joints has been investigated experimentally by Mason *et al.* (2010) on small-diameter (304.8mm – 12 in.) pipes with D/t ratio equal to 50, significantly thicker than the pipes used for water transmission. It was found that failure of the welded-lap joints occurred at strains higher than 2%, which indicates that those joints were capable of sustaining inelastic deformation before failure. Moreover, the experimental testing and finite element calculations on the compression strength of welded-lap connections (Tsetseni and Karamanos, 2007; Mason *et* *al.*, 2010), have indicated that for pipes with D/t ratio equal to about 100, welded-lap joint efficiency is close to 0.8, but reduces for pipes with higher values of D/t ratio. This efficiency value is significantly higher than the values suggested by the ASME B&PV (Boiler and Pressure Vessel) code, also noted by Smith (2006).

In a recent publication, the authors have examined the structural behavior of welded-lap joints in large-diameter pipes ($D/t =$ 150, 240), subjected to bending in the presence of internal pressure using three-dimensional nonlinear finite element models (Karamanos *et al.*, 2015b). It was found that the principal mode of failure is local buckling at the joint area. Furthermore, the results indicated that upon occurrence of local buckling, local strains may increase very rapidly in several critical locations. More recently, McPherson *et al.* (2016) have proposed a strengthening technique of welded-lap joints using a steel outer-bell, expanded in the pipe mill together with the bell of the parent pipe. Numerical calculations from three-dimensional finite element models have shown that the outer bell provides extra strength to the welded-lap joint and constitutes a promising and efficient joint strengthening solution for welded steel pipes constructed in geohazard areas.

On the other hand, the behavior of pipe fittings (e.g. mitered elbows, pipe junctions) under severe structural loading has received less attention. The reader is referred to a recent paper by the authors on the structural behavior of mitered bends, where the issues of bending flexibility, stress intensity and local buckling failure are addressed (Karamanos *et al.*, 2016). It is the authors' opinion that the mechanical behavior of pipe fittings subjected to severe ground-induced actions and their effect on steel pipeline response constitutes an open issue that requires further investigation.

A NOTE ON THE USE OF GASKETED JOINTS IN SEISMIC AREAS

The use of gasketed joints in steel pipelines constructed in seismic zones has raised quite some debate. Because of their ability to allow for a small amount of relative displacement and rotation between the two adjacent pipe segments, there exists an argument that supports the use of gasketed joints in seismic areas. More specifically, it has been argued that the relative motion of adjacent parts in gasketed joints may be able to accommodate ground-induced pipeline actions in an efficient manner. It is the authors' opinion though that, in the case of severe permanent ground deformations, the capability of a "segmental" pipeline with gasketed joints to sustain significant tensile loading is questionable, mainly because the corresponding displacement at the joints may localize at one joint, resulting in excessive local relative displacement and loss of pipeline continuity.

Furthermore, the behavior of gasketed joints under severe bending loading is an open issue. A recent work on the behavior of gasketed joints on 6-inch-diameter ductile iron pipes ($D/t=21$) has shown that those joints exhibited a substantial rotational capacity of 16 degrees (Wham and O'Rourke, 2016). However, steel pipes employed in steel pipeline applications, are much thinner than ductile iron pipes, and relative rotation due to severe bending will cause high local strains and deformations that may damage the pipe and the gasket leading to loss of containment. A dedicated investigation that combines experimental and numerical work is necessary, so that reliable deformation limits for gasketed joints subjected to bending in large-diameter steel pipes are determined.

On the other hand, it is expected that gasketed joints, properly designed, are capable of accommodating seismic transient effects, and therefore, they can be employed in seismic zones where severe permanent ground-induced actions are not expected. Following the provisions of ALA Guidelines (2005), the displacement Δ_{joint} that the gasketed joint should be able to sustain from transient seismic action is equal to

$$\Delta_{\text{joint}} = 7L_p\varepsilon_g + 0.25\,in \qquad (17)$$

where ε_g is the ground strain of equation (1) and L_p is the length of a pipe segment. In equation (17), the extra value of 0.25 in is considered as a factor of safety, and a factor equal to 7 is introduced, accounting for the uncertainly associated with the distribution of axial displacement in a segmental pipeline under tensile loading; the corresponding expansion may not be equally distributed in all gasketed joints, the deformation at one joint may localize, so that the two pipeline parts are separated, leading to loss of containment. The above design procedure is described in the Design Example in a later section. Finally, a fragility analysis of such joints under seismic wave loading can be found in the paper by O'Rourke *et al.* (2015).

MITIGATION MEASURES AGAINST SEISMIC ACTIONS

Several measures can be employed to mitigate seismic damage to pipelines. The most obvious action to minimize earthquake effects is the modification of pipeline alignment to avoid seismic and geo-hazard areas (pipeline re-routing). However, in the majority of cases, this may not be possible; therefore, specific mitigation measures should be adopted to minimize ground-induced strains in the buried pipeline. More specifically:

- The increase of pipeline wall thickness increases pipeline strength against seismic action. Both buckling and tensile resistance of the pipeline wall increase with increasing thickness.
- The use of higher grade line pipe material increases pipeline strength. However, one may be cautious for the reduced ductility of high-strength steel, usually expressed through the yield-

to-tensile strength ratio (Y/T); permanent ground actions are applied through a displacement-controlled scheme and – in such a case – material ductility and deformation capacity may be more important than strength.

- In areas where significant permanent ground deformations are expected, the designer may consider to isolate the pipeline from the ground movements, using either an above-ground pipeline section, appropriately supported in the ground, or a tunnel around the pipeline, so that the pipeline does interact with the surrounding soil.
- In landslide areas, it may be possible to improve ground conditions, using a slope drainage system, so that the risk of slop instability is reduced.
- In fault crossings, stiff soil conditions introduce higher stresses and strains in the pipeline. Therefore, the use of soft backfill soil would result in reduced stresses and strains within the pipeline. However, a soft cover may reduce its resistance in global buckling, and therefore, such a solution may be used cautiously.
- In strike-slip faults, the crossing angle should be such that the pipeline is in tension and not in compression. Based on recent finite element results (Vazouras *et al.* 2015), a crossing angle equal to 10-20 degrees appears to be an optimum angle for strike-slip faults.
- In fault crossing, the use of flexible components (e.g. elbows), may not be recommended within the fault zone. Nevertheless, in fault crossings, associated with significant pipeline tension, using elbows at an appropriate distance from the discontinuity area, may result in a reduction of axial stretching and the corresponding strains; the distance depends on elbow geometry, soil properties and the direction of the fault.
- Where possible, reverse vertical faults (thrust faults) should be avoided because they result in high compressive stresses, which may cause buckling of thin-walled steel pipes.
- Specialized expansion joints and/or deflectable joints can be used as mitigation devices to reduce axial stretching of the pipeline in permanent ground motion areas.

DESIGN EXAMPLE

A buried steel pipeline is considered for a seismic zone. Seismic activity consists of transient seismic wave action, characterized by peak ground acceleration and velocity equal to 0.30g and 76.2 cm/sec respectively, as well as by a seismic fault crossing the pipeline with left lateral strike-slip offset of 203 mm, together with normal offset along the fault surface of 899 mm as shown in Figure 6. The pipeline has diameter and thickness equal to 1,524 mm (60 in.) and 8.1 mm (0.319 in.) respectively, with material X42 steel grade (σ = 290 MPa) and is buried at a depth equal to one pipe diameter D with respect to the top side of the pipe. Soil conditions are cohesionless, with the following properties: density $\gamma = 17.4$ kN/m^3, friction angle $\varphi = 34^{\circ}$, and lateral earth pressure coefficient K_0 was assumed equal to 0.5. For the sake of simplicity, in this area the pipeline alignment is considered straight, without pipe bends. Pipeline behavior and design are outlined below, for both seismic wave action and fault movement.

Pipeline joint configuration

It is proposed that the steel pipeline will be welded (continuous), with welded lap joints, in the fault crossing area, and segmental, with gasketed joints, away from this area. The configuration of the gasketed joint is shown in Figure 7, whereas the welded-lap joints are considered double-welded (inside and outside weld) for maximizing their strength.

Therefore, a welded (continuous) pipeline should be considered in the analysis of permanent ground-induced fault action, whereas the analysis of seismic wave action should refer to a segmental pipeline. The former analysis should also determine the length of welded pipeline segment. These two analyses are briefly described below.

Seismic wave action

Seismic wave action on the pipeline is calculated from equation (17), in terms of the relative displacement Δ_S in a gasketed joint, assuming a line pipe length equal to 12.1 m (40 ft), and apparent velocity equal to 3,050 m/sec (10,000 ft/sec)) as follows:

$$\Delta_S = 7L_p\varepsilon_g = 7L_p\frac{PGV}{C} = 21\,mm\,(0.84\,in.) \qquad (18)$$

and the total seismic displacement is equal to

$$\Delta_{\text{joint}} = \Delta_S + 0.25\,in = 27.7\,mm\,(1.09\,in.) \qquad (19)$$

This displacement of 27.7 mm can be sustained by the gasketed joint under consideration shown in Figure 7.

Fault crossing analysis

Based on Figure 6, the three components of pipeline action with respect to the pipeline (axial, horizontal transverse, vertical transverse).can be computed from the following geometric equations:

$$\delta_{Ha} = d_H\cos\theta + d_N\sin\alpha\sin\theta \qquad (20)$$

$$\delta_{HT} = d_H\sin\theta - d_N\sin\alpha\cos\theta \qquad (21)$$

$$\delta_V = d_N\cos\alpha \qquad (22)$$

The values of δ_{Ha}, δ_{HT} and δ_{V} are equal to 346.3 mm (13.63 in.), 117.6 mm (4.63 in.) and 835.4 mm (32.89 in.), and correspond to the values of $PGD_{FH}\cos\theta$, $PGD_{FH}\sin\theta$ and PGD_{FV} respectively, in equations (5) - (8).

Subsequently, the axial (membrane) strain is computed from equation (7), with values of lengths L_H, L_V and L_V' equal to 15.75 m, 14.67 m and 6.06 m respectively (see also Figure 2). These values have been found applying the methodology proposed by Sarvanis and Karamanos (2016). Subsequently, the bending strain in the vertical plane is also computed from equation (8). The maximum strain value is equal to 3.46%, shown in Table 1. This strain within the 2%-5% range.

In addition to the above analytical calculations, this fault crossing has been analyzed using finite element models (level 1), which employ special purpose elements for the pipe ("elbow" elements) and nonlinear springs for the soil. Spring constants have been determined according to ALA Guidelines (2005) and are shown in Table 2. The distribution of axial strains is shown in Figure 8, and the maximum strain is equal to 3.09%, shown in Table 1, which is quite close to the value computed above using the analytical expressions (7) and (8), yet somewhat lower, which is beneficial for the pipeline. According to EN 1998-4 provisions, this value is very close to the specified limit (3%) and can be acceptable. However, according to ALA guidelines (2005) it may not be sustained by a welded-lap joint because it exceeds 2%. In such a case where mitigation measures are necessary, such as pipeline realignment or the use of a softer backfill, in an attempt to reduce ground-induced tensile strain. Furthermore, strengthening of the welded lap joints may increase pipeline resilience. In any case, it is authors' opinion that the development of reliable tensile strain limits for welded lap joints is necessary, and should be a research priority.

The numerical results also show that the high strains are developed in a small length of 10 m about the fault, whereas outside this zone, the strain level does not exceed the value of 0.2%. More specifically, pipeline stretching decays rapidly outside the fault zone and becomes negligible at a distance of approximately 250 m from the fault plane on either side of the fault. It is the authors' opinion that within this length, the pipeline should be welded (total length of welded pipeline equal to 500 meters), given also the uncertainty on the exact location of the fault. Outside this length, segmental (gasketed) joints can be employed.

SUMMARY AND CONCLUSIONS

Seismic design of buried steel water pipelines is a topic of significant importance for safeguarding the structural integrity of pipelines constructed in seismic zones. However, current pipeline design standards contain limited information on seismic design. The ALA (2005) guidelines, together with the Indian NICEE recommendations (2007), constitute documents on this subject that can be used for design purposes, whereas the PRCI (2004) refer mainly to hydrocarbon pipelines.

Soil-pipe interaction is the key issue for determining ground-induced strains on the pipe wall. For the case of permanent ground-induced actions, the designer may use a finite element model for efficient stress analysis of the pipeline. However, analytical expressions can be used to obtain reasonable estimates of ground-induced strains in the pipeline. Furthermore, the paper describes the main issues related to the mechanical behavior and pipe resistance of buried thin-walled welded steel pipelines, referring to the relevant failure modes. It is the authors' opinion that additional research is necessary to determine the strength and deformation capacity of pipeline joints and fittings under axial and bending loading.

At the end of the paper, the above design framework is applied in a specific case study that involves both permanent and transient seismic actions. It is shown that an appropriate combination of welded-lap and gasketed joints may offer a good solution for buried steel pipelines constructed in seismic zones.

ACKNOWLEDGEMENTS

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Analysis and Design of Buried Steel Water Pipelines in Seismic Areas

Spyros A. Karamanos, Gregory C. Sarvanis, Brent D. Keil and Robert J. Card

List of Tables

Table 1. Comparison between FE model and design equations.

Maximum Tensile strain (%)	**Analytical**	**FEM**
	3.46	**3.09**

Table 2. Maximum soil resistance and corresponding displacements according to ALA (2005).

Soil-Springs According to ALA (2005)	**Force per unit length of pipe (kN/m)**	**Corresponding displacement (m)**
Axial	52.9	0.005
Horizontal	544	0.090
Vertical - uplift	90	0.015
Vertical - bearing	2560	0.150

Figure 1

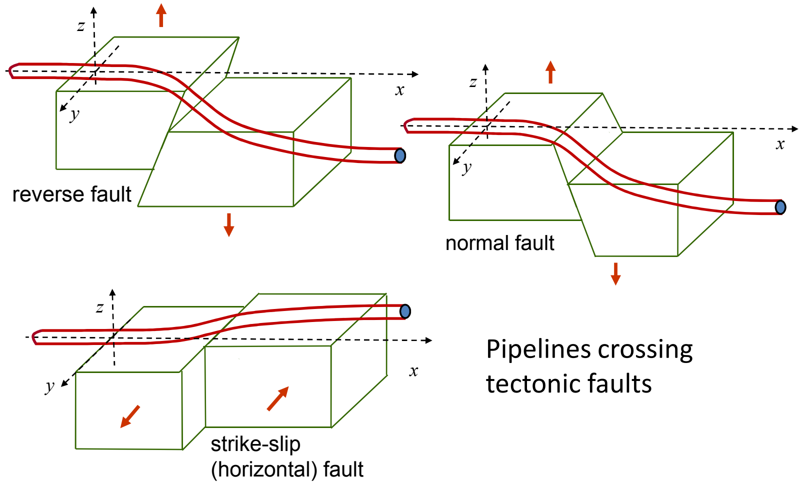
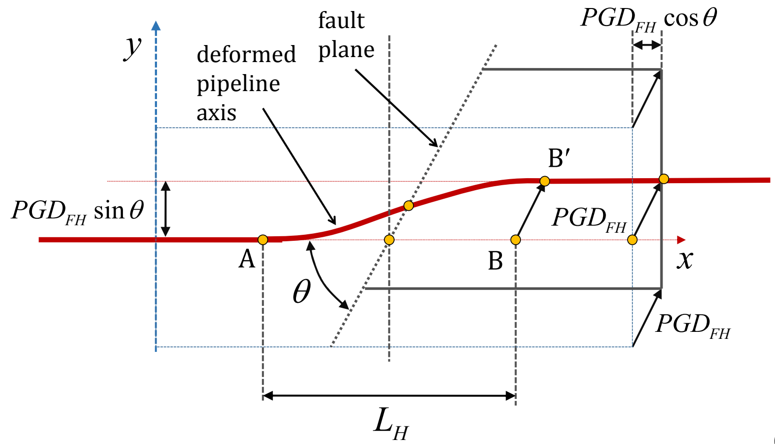
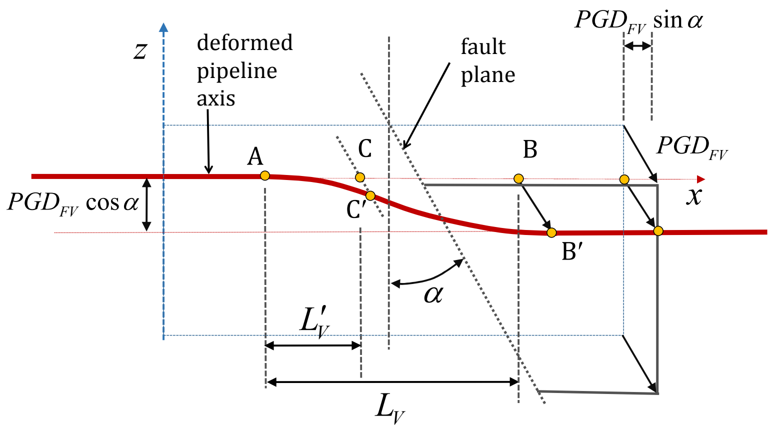

z
x
y
reverse fault
z
x
y
normal fault
z
x
y
strike-slip
(horizontal) fault
Pipelines crossing
tectonic faults

Figure 2



y
deformed
pipeline
axis
fault
plane
$PGD_{FH}\cos\theta$
B′
$PGD_{FH}\sin\theta$
PGD_{FH}
A
B
x
θ
PGD_{FH}
L_H

(a)



z
deformed
pipeline
axis
fault
plane
$PGD_{FV}\sin\alpha$
A
C
B
PGD_{FV}
x
$PGD_{FV}\cos\alpha$
C′
B′
α
L'_V
L_V

(b)

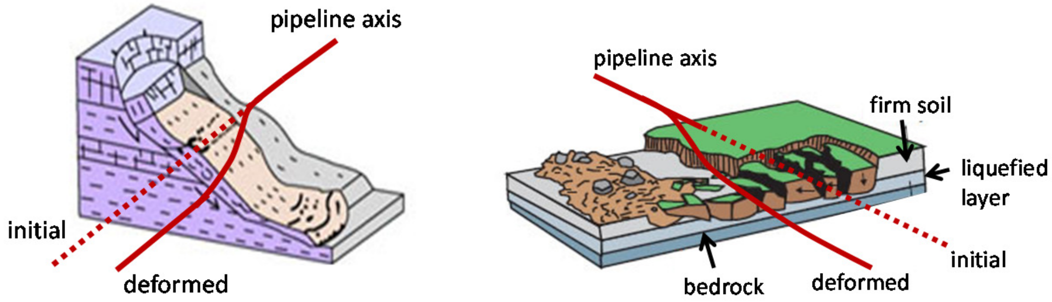

pipeline axis
initial
deformed
pipeline axis
firm soil
liquefied layer
initial
bedrock
deformed

Figure 4

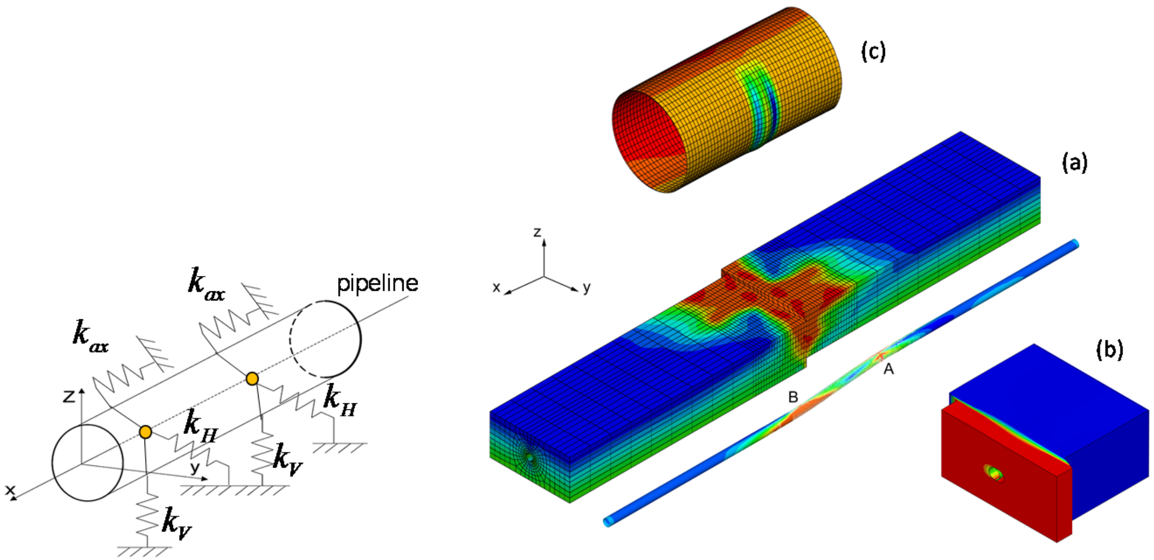
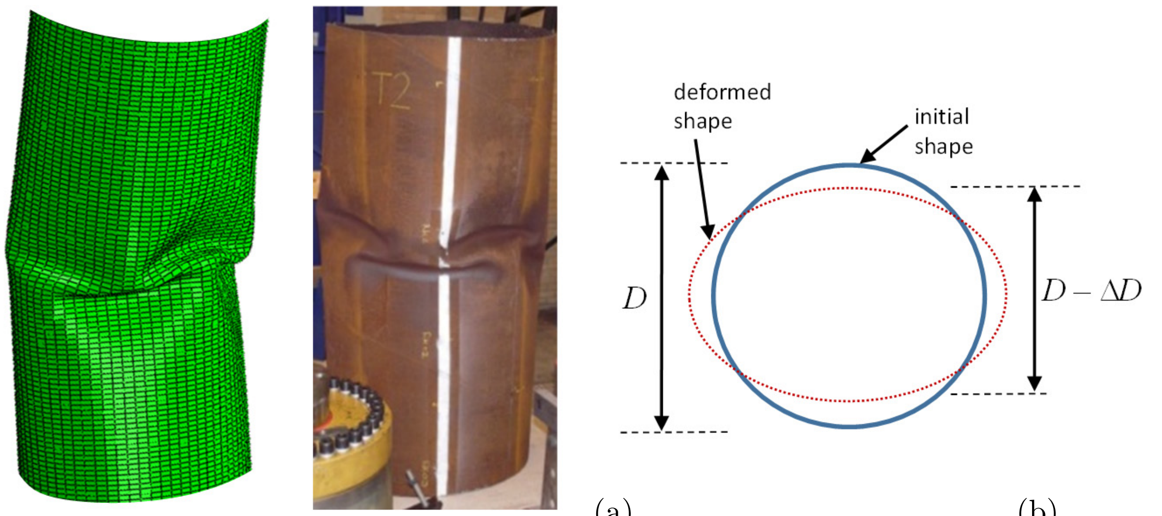
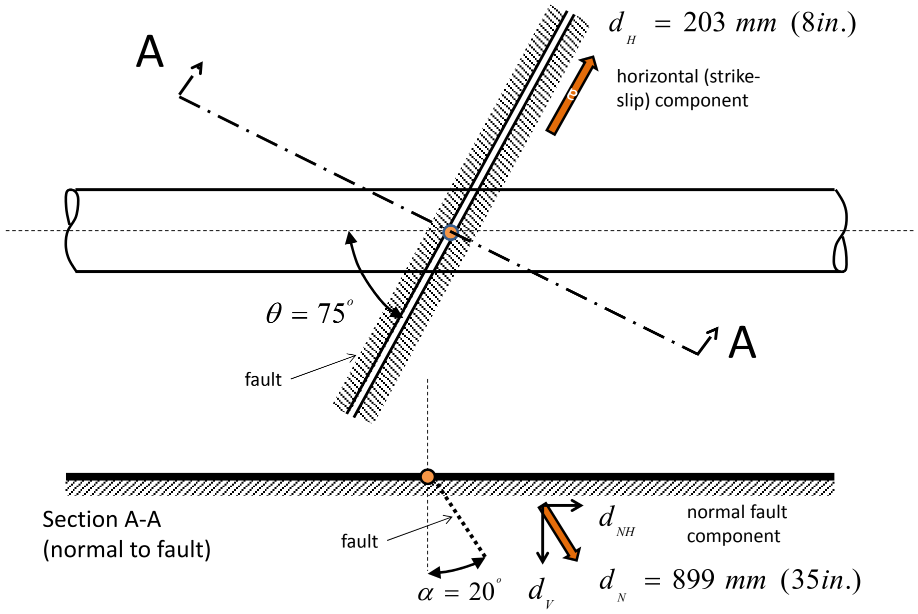
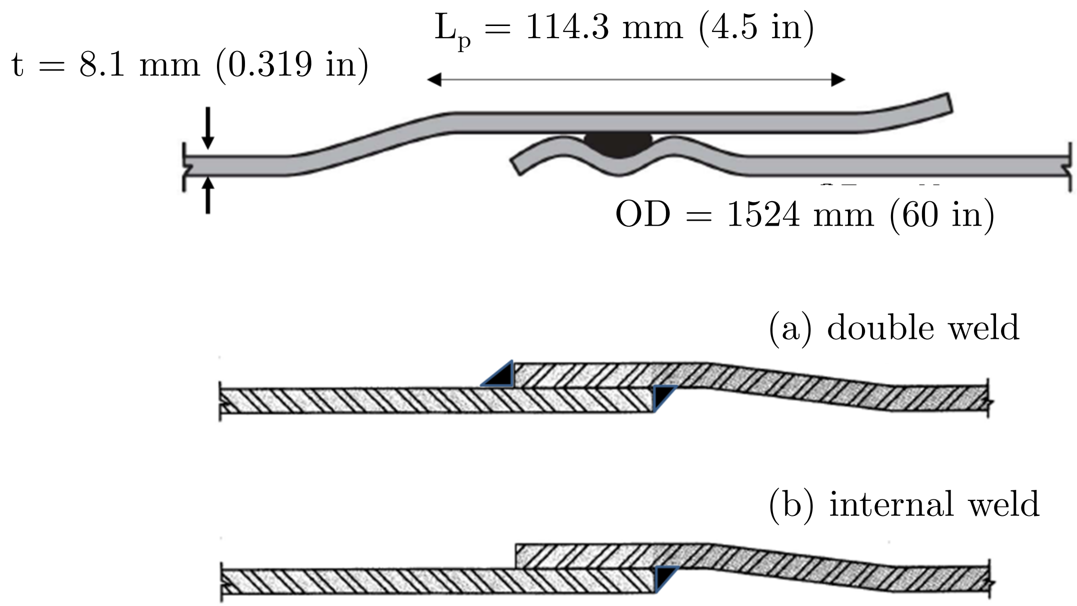


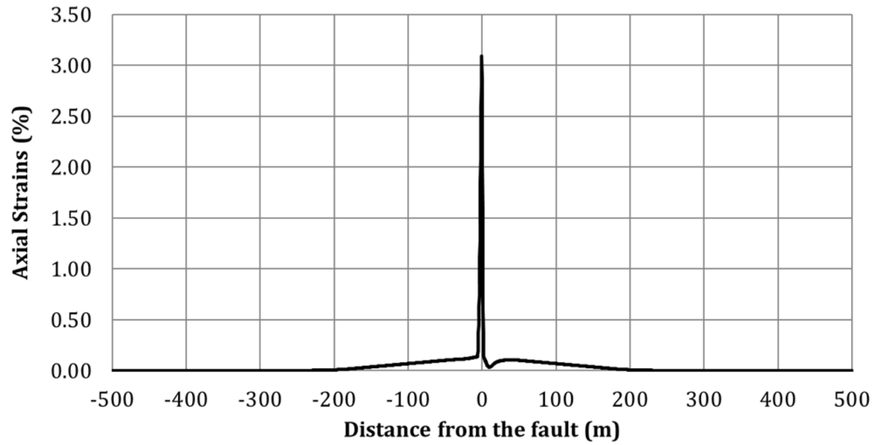
Figure 5 Click here to download Figure Figure 5.pdf



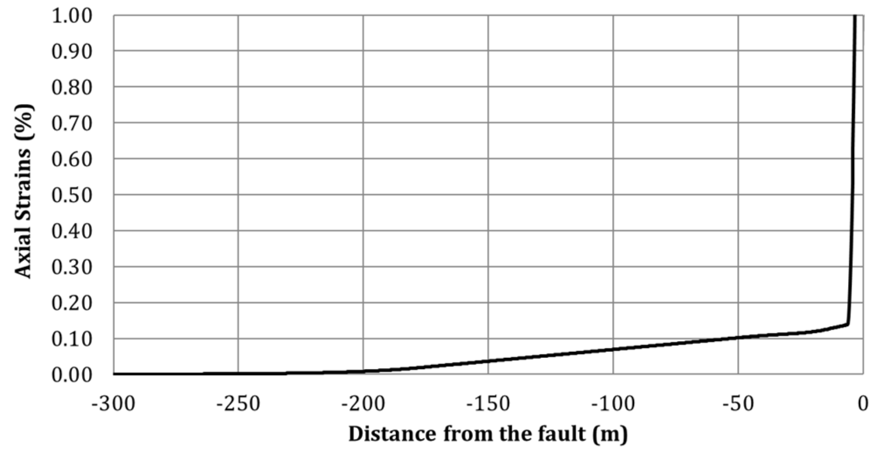
(a) (b)


$d_H = 203\ mm\ (8in.)$
horizontal (strike-slip) component
A
A
$\theta = 75^o$
fault
Section A-A
(normal to fault)
fault
$\alpha = 20^o$
d_V
d_{NH}
normal fault component
$d_N = 899\ mm\ (35in.)$


L_p = 114.3 mm (4.5 in)
t = 8.1 mm (0.319 in)
OD = 1524 mm (60 in)
(a) double weld
(b) internal weld


3.50
3.00
2.50
2.00
1.50
1.00
0.50
0.00
Axial Strains (%)
-500 -400 -300 -200 -100 0 100 200 300 400 500
Distance from the fault (m)

(a)


1.00
0.90
0.80
0.70
0.60
0.50
0.40
0.30
0.20
0.10
0.00
Axial Strains (%)
-300 -250 -200 -150 -100 -50 0
Distance from the fault (m)

(b)

Analysis and Design of Buried Steel Water Pipelines in Seismic Areas

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Figure 1: Schematic representation of pipeline configuration crossing tectonic faults.

Figure 2: Pipeline deformation crossing: (a) a horizontal fault at angle θ (plan view); (b) a normal fault at angle α (side view).

Figure 3: Schematic representation of pipeline configuration in the boundary of landslide (left) and liquefaction-induced lateral spreading (right) [source: USGS; http://pubs.usgs.gov/].

Figure 4: Level 1 of pipeline modelling; pipe (beam-type) finite elements and soil springs attached to pipeline nodes in the three principal directions (left); Level 2 of pipeline modelling; shell elements and solid elements (right) [Vazouras *et al.*, 2010].

Figure 5: (a) Local buckling of a spiral-welded pipe with $D/t = 119$ due to excessive pipe wall compression, subjected to longitudinal bending; unpublished numerical simulation conducted at the University of Thessaly, and test conducted at TU Delft, the Netherlands [Van Es *et al.* 2016, Vasilikis *et al.* 2016]. (b) Definition of pipe cross-sectional flattening.

Figure 6: Pipeline crossing a seismic fault in a seismic zone; (a) plan view of pipeline crossing and (b) section A-A.

Figure 7: Configuration of (top) gasketed joint used in the pipeline; (bottom) double-welded and internally-welded lap joints.

Figure 8: Axial strains along the pipeline, at the fault crossing area; (a) strain distribution along a long pipe segment; (b) strain distribution up to 300 m from the fault.

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Response to reviewers

- There were no comments made by the reviewers at this stage ("accept as is").
- All editorial requests have been addressed in the present (final) submission.
- We would like to thank the editor and the reviewers for reviewing and approving the publication of this manuscript.